

## **A NON-LINEAR MASONRY INFILL MACRO-MODEL TO REPRESENT THE GLOBAL BEHAVIOUR OF BUILDINGS UNDER CYCLIC LOADING**

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### **SYNOPSIS**

The presence of masonry infill walls in RC buildings is very common. However, even nowadays, in the design of new buildings and in the assessment of existing ones, the infills are usually considered as non-structural elements and their influence in the structural response is ignored. For horizontal loading, infill panels can drastically modify the response, attracting forces to parts of the structure that have not been designed to resist them (Varum, 2003). In this paper is presented an improved non-linear numerical model for the simulation of the influence of the masonry infill walls in the seismic behaviour of structures, implemented in the analysis program PORANL.

After the implementation and calibration of the proposed masonry model, it was performed a series non-linear dynamic analyses of a building, representative of the Modern Architecture. The main objective was to investigate the behaviour of this type of buildings, and their weakness under seismic loading. The building geometry and dimensions of the RC elements and infill walls were given in the original project, and were confirmed in the technical visits. The building studied has nine storeys and the structure is mainly composed by twelve plane frames oriented in the transversal direction. The building was analysed with a simplified plane model, for each direction, and the existing infill panels were considered according to their dimensions and location. The earthquake action adopted in this study was simulated with three series of artificially generated earthquakes, for a medium/high seismic risk scenario in Europe.

### **INFLUENCE OF INFILL MASONRY WALLS ON THE STRUCTURAL SEISMIC RESPONSE**

It is incorrect to assume that masonry infill panels are always beneficial to the structural response. The contributions of the infills for the building seismic response can be positive or negative, depending on a series of phenomena and parameters as, for example, relative stiffness and strength between the frames and the masonry.

In recent earthquakes, numerous buildings were severely damaged or collapsed due to the structural modifications of the basic structural system induced by the non-structural masonry partitions (Fig. 1). Even if they are relatively weak, masonry infill walls can drastically modify the structural response.

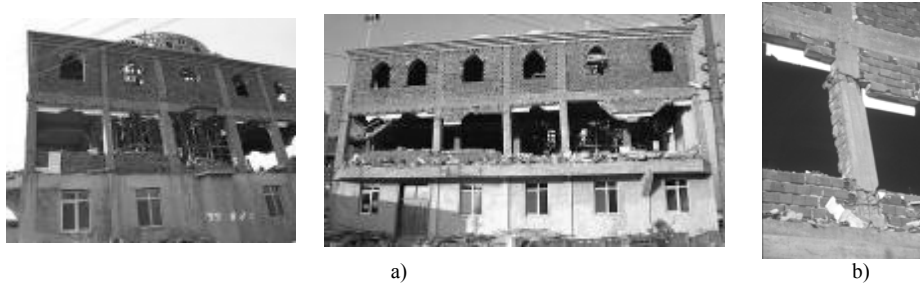


Fig. 1 – Damage on masonry infill walls

Masonry infill panels can increase substantially the global stiffness of the structure. Consequently, its natural period will decrease. Depending on the situations in which masonry walls extend, for example only to part of the storey-height (short-columns) leaving a relatively short portion of the columns exposed, may also induce vulnerable behaviour. Frequently, a column is shortened by elements which have not been taken into account in the global structural design, for example, window openings or landing slabs of staircases (Varum, 2003).

### PROPOSED MASONRY INFILL PANEL'S MODEL

The infill masonry models can be classified as micro and macro models. In the micro-models, infill panels are modelled with detailing at components level: mortar, bricks, and interface mortar/brick elements. With the micro-models, a more accurate representation of the infill panels' behaviour can be obtained. However, an enormous calculation effort and a large amount of parameters have to be calibrated. They can be useful for local analysis, but impractical for the global analysis of a building (Rodrigues, 2005).

The macro-models are more simplified and allow for the representation of the infill panel' global behaviour and its influence in the building structural response. The macro-model most commonly used is the bi-diagonal equivalent-strut model. Many examples of other macro-models can be found in the literature: i) homogenized frames sections; ii) theory of plasticity; iii) behaviour coefficients, among many others.

The proposed macro-model is an improvement of the commonly used equivalent bi-diagonal-strut model. The proposed model considers the interaction of the masonry panel' behaviour in the two directions. To represent a masonry panel are considered: four support strut-elements with rigid-linear behaviour; and, a central element where the non-linear hysteretic behaviour is concentrated (Fig. 2-a).

The non-linear behaviour is characterized by a multi-linear envelop curve, defined by nine parameters (Fig. 2-b), representing: cracking, peak strength, stiffness decreasing after peak strength and residual strength, for each direction, which allows for the non-symmetrical behaviour consideration. The hysteretic rules calibrated for masonry models are controlled by three additional parameters, namely:  $\alpha$  - stiffness degradation;  $\beta$  - "pinching" effect; and,  $\gamma$  - strength degradation.

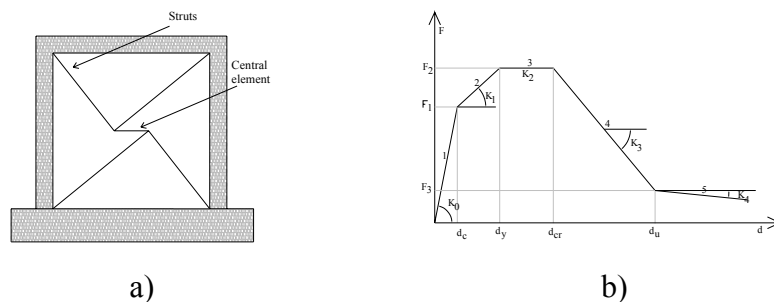


Fig. 2 – Implemented strut model and force-displacement monotonic behaviour curve

The non-linear behaviour of the central element is characterized by hysteretic rules based on the Takeda's model (Costa, 1989), illustrated in Fig. 3, allowing to determine the response to the loading history depending on the material's behaviour (defined by the envelop curve and hysteretic parameters), and represents mechanical effects as stiffness and strength degradation, pinching effect, and internal cycles (Rodrigues *et al.*, 2005).

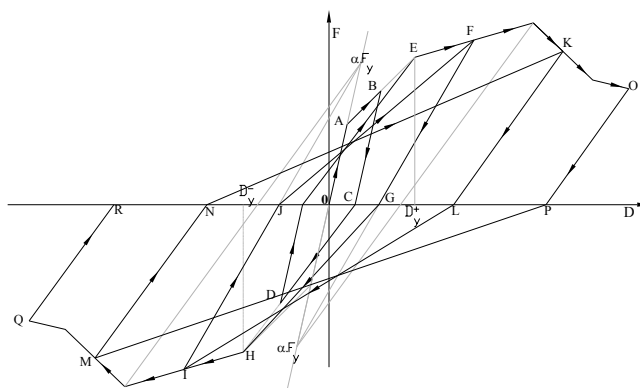


Fig. 3 – Hysteretic rules for the implemented model

The monotonic behaviour curve of each panel depends on the panel dimensions, eventual openings dimensions and position, material properties (bricks, mortar, and plaster), quality of the handwork, interface conditions between panel and surrounding RC elements, and can be obtained from empirical expressions or experimental results (Zarnic and Gotic, 1998).

## RC ELEMENTS

Nowadays, in the analysis of structures subjected to seismic actions, the use of non-linear behaviour laws and hysteretic rules reveals a great advantage, because it makes possible a more rigorous representation of the seismic structural response.

To simulate the structural behaviour of the building presented in the previous section, it was used a computer program PORANL, that contemplates the non-linear bending behaviour of RC elements (beams and columns) and the influence of the infill masonry panels.

Each RC structural element is modelled by a macro-element defined by the association of three bar finite elements, two with non-linear behaviour at its extremities (plastic hinges), and a central element with linear behaviour, as represented in Fig. 4.

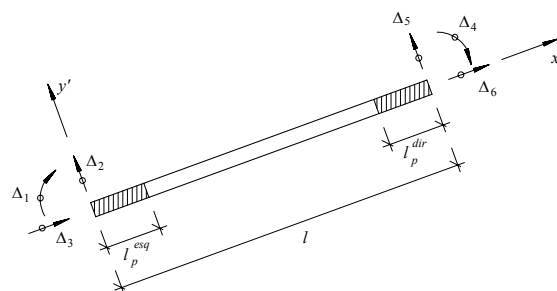


Fig. 4 – Frame macro-element

The non-linear monotonic behaviour curve of a cross-section is characterized through a tri-linear moment-curvature relationship, corresponding to the initial non-cracked concrete, concrete cracking and steel reinforcement yielding (Varum, 1996). The monotonic curve is obtained using a fibre model procedure (see Fig. 5), from the geometric characteristics of the cross-sections, reinforcement and its location, and material properties.

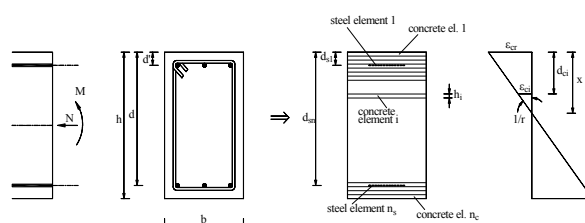


Fig. 5 – General fibre model for RC elements

The non-linear behaviour of the plastic hinge elements is controlled through a modified hysteretic procedure, based on the Takeda model, as illustrated in Fig. 6. This model developed by Costa (1989) represents the response evolution of the global RC section to seismic actions and contemplates mechanical behaviour effects as stiffness and strength degradation, pinching effect, slipping, internal cycles, etc.

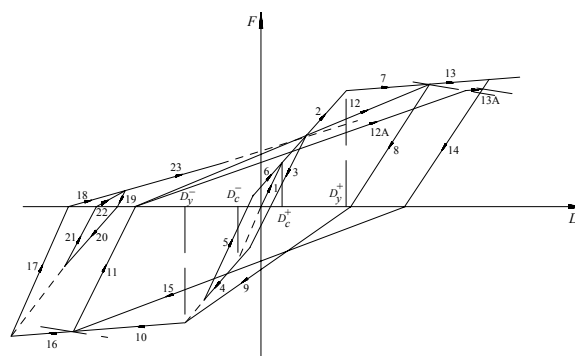


Fig. 6 – Hysteretic model for RC elements

## CASE STUDY

The development and calibration of refined numerical tools, as well as, assessment and design codes makes feasible the structural safety assessment of existing buildings. To investigate the vulnerability of this Lisbon common type of buildings, one representative of the Modern Architecture, at the Infante Santo Avenue, was studied. This building was studied with the non-linear dynamic analysis program PORANL, which allows the safety evaluation according to the recently proposed standards.

The 3<sup>rd</sup> parcel from the Unity Type A in the Infante Santo Avenue, in Lisbon, is a singular example of Modern Housing Project in Portugal (see Fig. 7). It's a building inserted in an Urban Plan initialized by Alberto José Pessoa (1919-1985), in 1947, when he joined the "Urbanization Study on the Protection Area of Palácio das Necessidades" in Câmara Municipal of Lisbon (CML). It consisted on dividing and expropriating land parcels for the new Infante Santo Avenue, which completed the ring created by De Gröer's Municipal Plan (1938-1948).

The block plan is rectangular with 11.10m width and 47.40m length. The building has the height of 8 habitation storeys plus the pilotis height at the ground floor. The "free plan" is also a reference because the house was conceived in a way of flexibility in use. But, the 12 structural plane frames define the architectural plan of the floor type, with 6 duplex apartments. The distance between frame's axes is 3.80m. Each frame is supported by two columns and has one cantilever beam on each side with 2.80m span, resulting in 13 modules.



Fig. 7 – General views of the building block under analyses

The structural plan presents an adequate solution to the architecture's objectives. "From the beginning of the studies, it was a permanently concern, to conceive a resistant structure concept, simple, elegant and economic. And it looks like the objective was well succeeded because since the beginning of the project elaboration there wasn't any need for changing the primitive structure".

The structural plan for the housing block of the Unity Type A is comprised by 12 transversal reinforced concrete frames, formed each one by two columns and 3 beams at each storey, two of them in cantilever.

The structural design was initially made just for vertical loads, without considering the columns bending moment. Afterwards, new designs were developed, now considering in a simplified way the horizontal loads, corresponding to the wind, using the Cross's method in the bending moments distribution calculation. J. V. Dias does not considered in its design the seismic action. He refers "the low probability of simultaneous occurrence of wind and seismic actions in the same direction and at their maximum intensity". Dias concludes that "this building-type has superior safety conditions than the majority of Lisbon's buildings".

Later, the importance of considering seismic actions in structural element's design was recognized. It was delivered a new design project according to an article of Maria Amélia Chaves and Bragão Farinha published in "Técnica" Magazine. Horizontal forces, proportional to the floor's mass were considered in the frame nodes. But, the structural analysis was only made in the transversal direction. The structural engineer concludes that wind forces induce larger demands than seismic loads, resulting in larger cross-sections.

The Engineer Ramos Cruz, responsible for the construction, did a new design project for the 3<sup>rd</sup> parcel. He presented new calculations based on the primitive project, but he changed the original structural floor by a reinforced concrete slab. He advocates that with this continuous

rigid slab, rigid diaphragm behaviour is guaranteed. Another modification is the introduction of reinforced concrete walls in the staircases at the ground floor, one of them was continuous in all building's height. However, these RC elements were not detected in the technical visit to the building (Miranda *et al.*, 2005).

## DESCRIPTION OF THE STUDIED STRUCTURE

The building geometry and dimensions of the RC elements and infill walls were given in the original project (1950-1956), and were confirmed in the technical visits (Miranda *et al.*, 2005). As already presented in a previous section, the building under study has nine storeys and the structure is mainly composed by twelve plane frames oriented in the transversal direction (direction Y, as represented in Fig. 8). The building was analysed with a simplified plane model for each direction (X - longitudinal direction, Y - transversal direction).

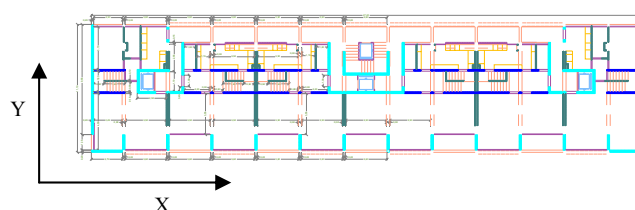


Fig. 8 – Structural system (plan)

The twelve transversal plane frames have the same geometric characteristics for all beams and columns. However, three different frame-types (labelled A, B and C) were identified, according to reinforcement detailing differences.

A peculiar structural characteristic of the type of buildings, with direct influence in the global structural behaviour, is the ground storey without infill masonry walls. Furthermore, at the ground storey the columns are 5.5m height. All the upper storeys have an inter-storey height of 3.0m.

In the two structural models (X and Y) a concrete slab 1.25m width and 0.20m thick. A detailed definition of the existing infill panels were considered in the structural models.

For the building analysis in the transversal direction (Y), it was assumed an equivalent model defined as the association of the three frame-types, interconnected by rigid strut bars, as showed in Fig. 9. In this global model, the geometric and mechanical characteristics of each frame are multiplied by the number of occurrences of each frame-type.

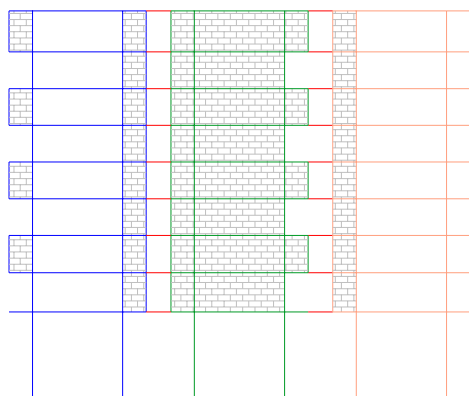


Fig. 9 – Equivalent structural system for transversal direction (Y)

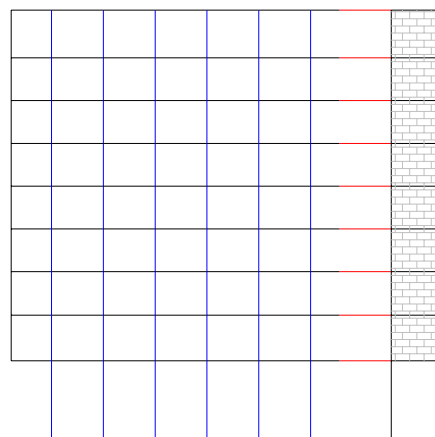


Fig. 10 – Equivalent structural system for longitudinal direction (X)

For the analysis in the longitudinal direction (X), and because the double symmetry in plan, it was studied just one quart of the building. For the global model results a six columns structure linked at all storey levels by the RC slabs. No full-bay infill panels exist in the longitudinal direction. Therefore, an external simplified global infill masonry model was considered, as represented in Fig. 10, connected throw rigid struts to the RC structure.

## STATIC LOADS, MASSES AND DAMPING

For the numerical analyses, constant vertical loads distributed on beams were considered in order to simulate the dead load of the self-weight including RC elements, and infill walls, finishing, and the correspondent quasi-permanent value of the live loads, totalising a value of  $8.0\text{kN/m}^2$ .

The mass of the structure was assumed concentrated at storey levels. Each storey has a mass, including the self-weight of the structure, infill walls and finishings, and the quasi-permanent value of the live loads, of about 4Mtons. For the dynamic analysis, the storey mass is assumed to be uniformly distributed on the floors.

A viscous damping ratio of 1% was considered in the numerical analysis for each vibration mode. This value is smaller than those normally used in the linear dynamic analyses (Varum, 2003).

For each structural model, a Rayleigh damping matrix, with 1% damping ratio for the first two natural modes, was considered, according with

$$[C] = \beta[K] + \alpha[M]$$

where the coefficients  $\beta$  and  $\alpha$  are calculated such that 1% damping ratio in the first two modes of vibration is achieved.  $[K]$  and  $[M]$  are the stiffness and mass matrices of the structure, respectively.

## NATURAL FREQUENCIES AND MODES

A first validation of structural numerical models can be achieved comparing the experimentally measured and the analytically estimated natural frequencies.

In Table 1 are listed the four first natural frequencies computed, for each building direction.

To validate the numerical building models, in the two independent directions, it were measured the first natural structural frequency, with a seismograph and the ambient vibration. The measured first frequency is indicated in brackets in Table 1.

TABLE 1: Natural frequencies for directions X and Y

Modes	Frequencies (Hz)	
	Longitudinal direction (X)	Transversal direction (Y)
1 <sup>st</sup>	1.08 (1.17)	1.75 (1.56)
2 <sup>nd</sup>	5.67	6.41
3 <sup>rd</sup>	6.32	8.14
4 <sup>th</sup>	8.10	8.80

A good agreement was found between the experimentally measured frequencies (1.17Hz for longitudinal direction and 1.56Hz for transversal direction - Miranda *et al.*, 2005) and the

frequencies estimated with the structural numerical models (1.08Hz for longitudinal direction and 1.75Hz for transversal direction), which constitutes the first validation of the numerical model. In Fig. 11 are represented the first natural mode for each direction.

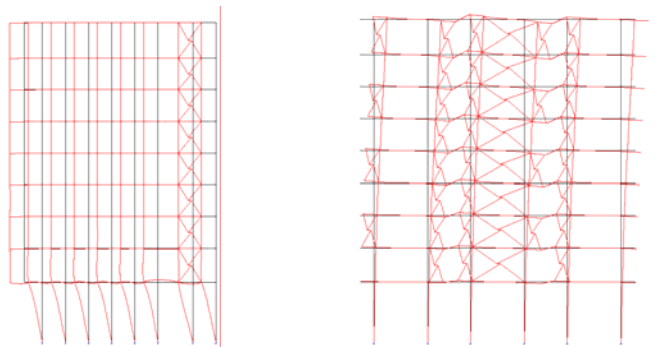


Fig. 11 – Natural vibration modes ( $f_{1,X} = 1.08\text{Hz}$  and  $f_{1,Y} = 1.75\text{Hz}$ )

From the analysis of the first vibration shape modes, in both directions, it is clear that the seismic structural response will induce soft-storey mechanism behaviour. This conclusion will be confirmed with the earthquake analysis results in the next sections.

## EARTHQUAKE INPUT SIGNALS

Three artificial earthquake input series were adopted for the seismic vulnerability analysis of the building. The first series (A) was artificially generated for a medium/high seismic risk scenario in Europe (Carvalho *et al.*, 1999), for various return periods (Table 2). The second and third series (B and C, respectively) were generated with a finite fault model for the simulation of a probable earthquake in Lisbon (Carvalho *et al.*, 2004), calibrated with real seismic actions measured in the region of Lisbon. The earthquakes of the B and C series were scaled to the peak ground acceleration of series A, for each return period. In Table 2 are presented the peak ground acceleration and the corresponding return period for each earthquake's intensity.

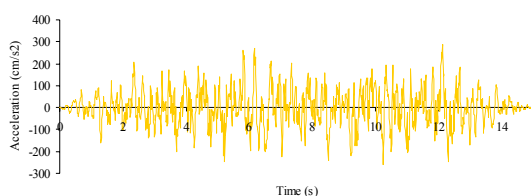


Fig. 12 – Accelerogram A

TABLE 2: Reference earthquake action (peak ground acceleration and corresponding return period)

Return period (years)	Peak ground acceleration ( $\times g$ )
73	0.091
475	0.222
975	0.294
2000	0.380
3000	0.435
5000	0.514

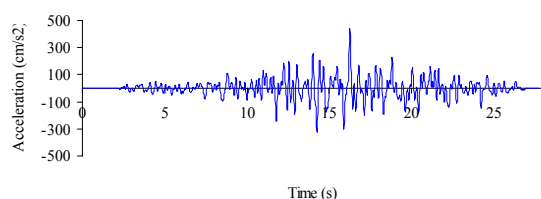


Fig. 13 – Accelerogram B

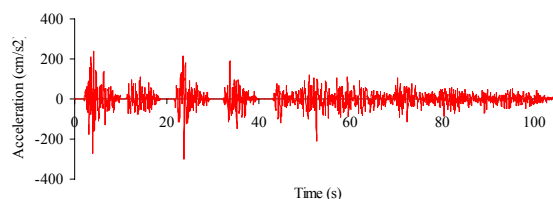


Fig. 14 – Accelerogram C



## RESULTS ANALYSIS

As observed in the analysis of the vibration shape modes, the structural response of the building, in both directions, clearly induces soft-storey mechanism behaviour (at the ground floor level). This structural behaviour leads to large inter-storey drift demands at the first storey, while the upper storeys remain with very low deformation levels.

In Fig. 15 and 16 are illustrated, for the longitudinal and transversal direction, respectively, the numerical results in terms of envelop deformed shape, maximum inter-storey drift, and maximum storey shear, for each earthquake input motion of the series A (73, 475, 975, 2000, 3000, 5000 years return period).

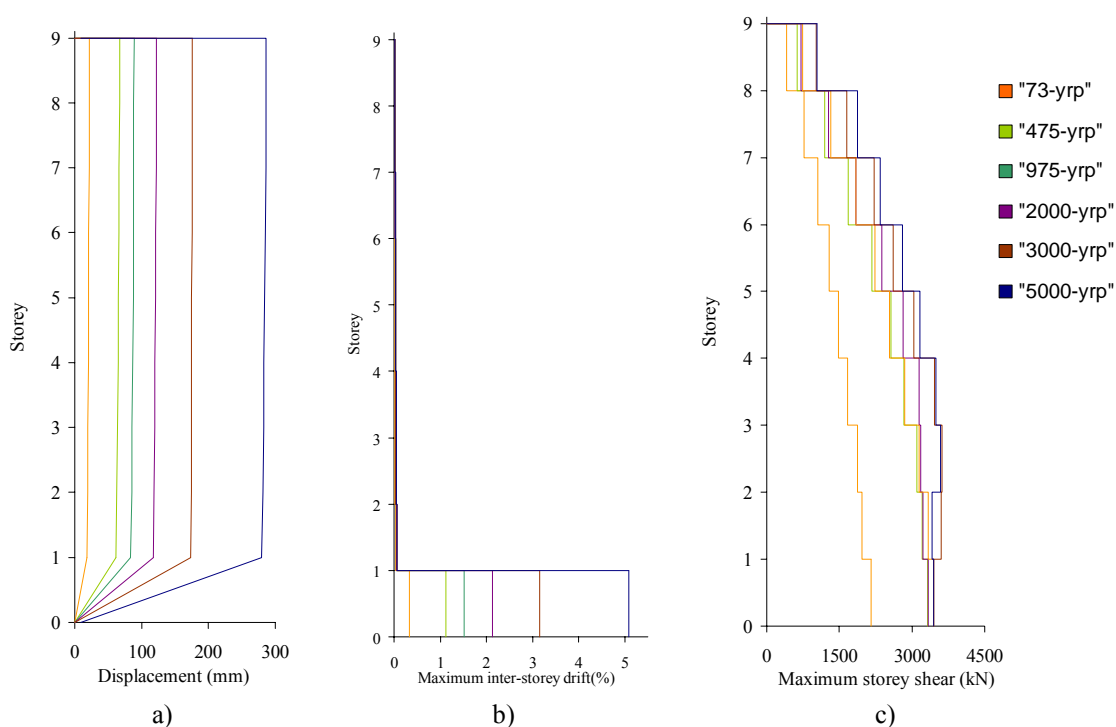


Fig. 15 – Results for the longitudinal direction (X) and earthquakes of the series A: a) envelop deformed shape; b) maximum inter-storey drift profile; c) maximum storey shear profile

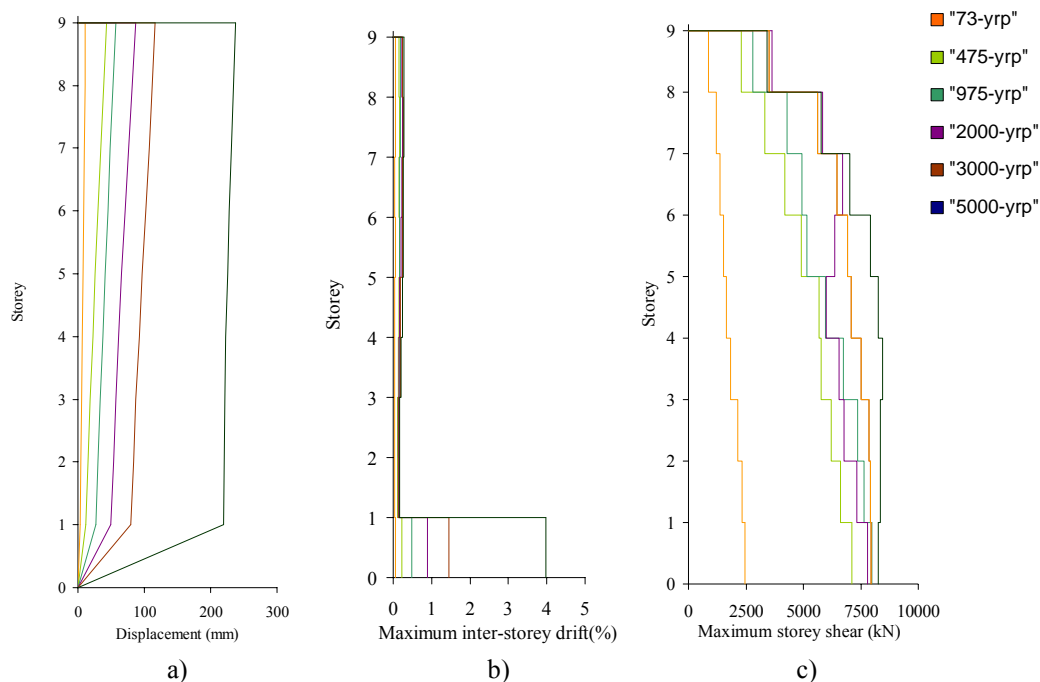


Fig. 16 – Results for the transversal direction (Y) and earthquakes of the series A: a) envelop deformed shape; b) maximum inter-storey drift profile; c) maximum storey shear profile

From the analysis of the results in terms of building envelop deformed shape and inter-storey drift profile, for both directions, it can be concluded that the deformation demands are concentrated at the first storey level. In fact, the absence of infill masonry walls at the ground storey and the larger storey height (5.60m for the 1<sup>st</sup> storey and 3.00m for the upper storeys), induces an important vertical structural irregularity, in terms of stiffness and strength.

For all the structural elements (columns and beams), and for all the seismic input action levels, the shear force demand assumes a value inferior to the corresponding shear capacity, which confirms its safety in shear.

### *Vulnerability curves*

In this section are compared, for the three earthquakes input motions, the vulnerability curves in terms of maximum 1<sup>st</sup> storey drift, maximum 1<sup>st</sup> storey shear and maximum top displacement, for the longitudinal and transversal directions.

In Fig. 17 and 18 are compared the vulnerability curves, for the longitudinal and transversal directions, for the maximum 1<sup>st</sup> storey drift, obtained from the numerical analysis. Results show that, for the 1<sup>st</sup> storey, the maximum inter-storey drift demand for the longitudinal direction is larger than for the transversal, being the first one the most vulnerable direction of the building.

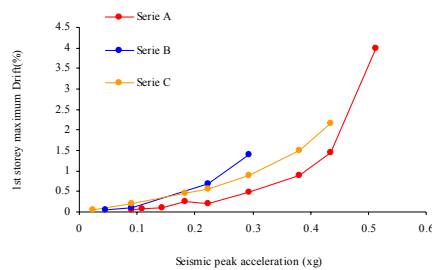


Fig. 17 – 1<sup>st</sup> storey drift vs. peak acceleration (transversal direction - Y)

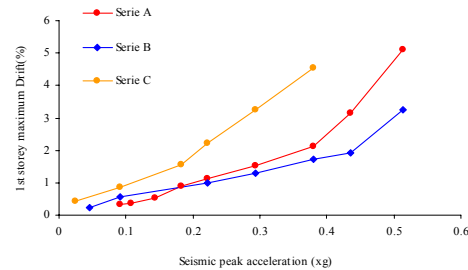


Fig. 18 – 1<sup>st</sup> storey drift vs. peak acceleration (longitudinal direction - X)

In Fig. 19 and 20 are represented the vulnerability curves in terms of maximum 1<sup>st</sup> storey shear force. In Fig. 21 and 22 are represented the obtained vulnerability curves in terms of maximum top displacement. Shear demand at 1<sup>st</sup> storey does not increase for earthquake input actions larger than the one corresponding to the return period of 475 years, inducing only increasing of deformation demands, as can be observed in the results in terms of 1<sup>st</sup> storey drift and top displacement.

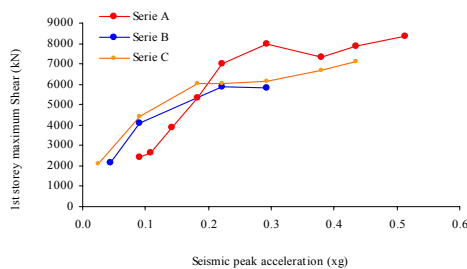


Fig. 19 – 1<sup>st</sup> storey shear vs. peak acceleration (transversal direction - Y)

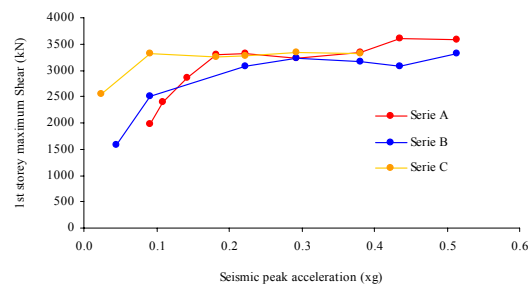


Fig. 20 – 1<sup>st</sup> storey shear vs. peak acceleration (longitudinal direction - X)

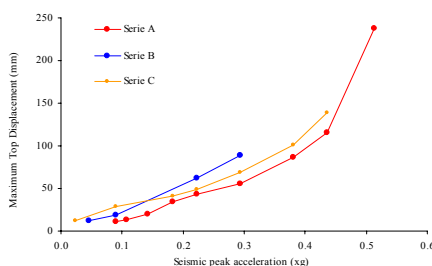


Fig. 21 – Top displacement vs. peak acceleration (transversal direction - Y)

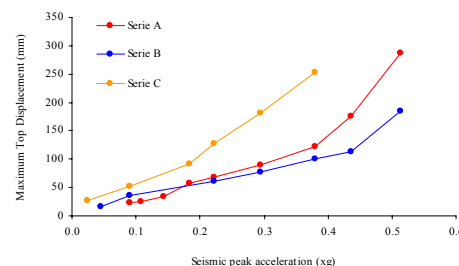


Fig. 22 – Top displacement vs. peak acceleration (longitudinal direction - X)

### Seismic safety assessment of the building

As presented in the previous section, for each direction, the building structure was analysed to three series of earthquakes with increasing intensities, in order to estimate deformation demands, and consequently damage levels.

The obtained results allow verifying the safety according to the hazard levels specified, for example, in VISION-2000 (SEAOC, 1995) and ATC-40 (1996) documents. In Tables 3 and 4

are presented the acceptable drift limits, for each structural performance level, according to the ATC-40 and in VISION 2000 proposals, respectively.

TABLE 3: Storey drift limits according to the ATC-40 (1996)

	Performance Level			
	Immediate Occupancy	Damage Control	Life Safety	Structural Stability
Drift Limit	1%	1-2%	2%	$0.33 \frac{V_i}{P_i} \approx 7\%$

TABLE 4: Storey drift limits according to the VISION-2000 (SEAOC, 1995)

	Performance Level			
	Fully Operational	Operational	Life Safe	Near Collapse
Drift Limit	0.2%	0.5%	1.5%	2.5%

In Fig. 23 and 24 are represented the vulnerability functions in terms of maximum 1<sup>st</sup> storey drift, presented in the previous section, with indication of the safety limits proposed at the ATC-40 and VISION-2000 recommendations (as summarised in Tables 3 and 4, respectively).

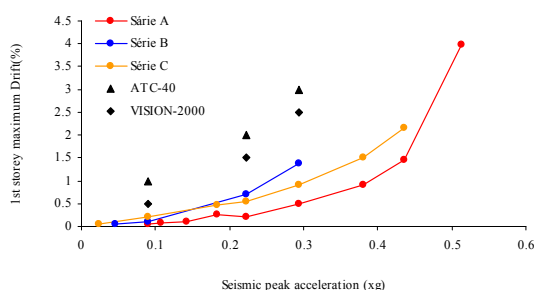


Fig. 23 – 1<sup>st</sup> storey drift vs. peak acceleration and safety limits (transversal direction - Y)

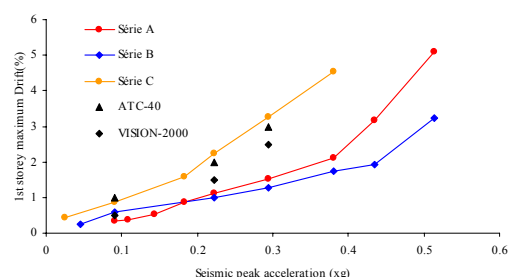


Fig. 24 – 1<sup>st</sup> storey drift vs. peak acceleration and safety limits (longitudinal direction - X)

Comparing the maximum storey drift demands with the safety limits proposed at the ATC-40 and VISION-2000 recommendations, it can be concluded that the building safety is guaranteed the transversal direction (Y), for the three earthquake input series considered. For the longitudinal direction (X), the safety is guaranteed for series A and B, but not for C series.

## CONCLUDING REMARKS

The global structural safety of a modern architecture style building in Lisbon, Portugal, was investigated. The analysis performed demonstrates the capacities of the models and of the integrated computer program, PORANL, in the representation of the global response and seismic behaviour of RC building structures. Particularly, the non-linear bending behaviour of

slender RC elements, and the participation of the infill masonry panels in the global seismic response of buildings was numerically simulated.

Although the first numerical results generally indicate the building safety for the Basic Safety Objectives, according to the international seismic recommendations (ATC-40 and VISION-2000), it should be pointed out that additional analyses have to be performed to validate this first conclusions.

The input motion earthquakes adopted for these analyses can be not fully representative of all the probable earthquake actions in Lisbon. Additional analyses should be performed using other earthquake input motions.

The model adopted for these analyses does not consider the geometric non-linearity, which can increase significantly the moments in columns and global storey lateral deformations (drifts). Therefore, to guarantee the seismic safety verification of the building, it is judged focal to verify the results using a model that considers the geometrical non-linearities.

It is evident the high vulnerability of this building structural typology, which exist in a considerable number in Lisbon region. The high seismic risk associated to these buildings in the Lisbon metropolitan area can be significantly reduced with the adoption of adequate retrofitting solutions. The seismic retrofitting of these buildings can be performed adopting economic solutions, since intervention can be resumed at the ground storey, usually clear, in this typology, for example with bracing systems and, eventually, combined with energy dissipation devices. After the building assessment and design of a retrofitting solution, its efficiency must be always evaluated.

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